1821 Almaden
Geology + Soils.

Project No. 3073
18 February 2013

Mr. Sanjeev Acharya
Silicon Valley Builders, LLC.
3333 Bowers Avenue, #130
Santa Clara, CA 95054

Subject: **PRELIMINARY GEOTECHNICAL INVESTIGATION**
Proposed Residential and Retail Stores Mixed Development
1821-1873 Almaden Road
San Jose, California

References:
1. Guidelines for Evaluating and Mitigating Seismic Hazards in California Special Publication 117A, Division of Mines and Geology, 2008
2. Recommendation Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing Landslide Hazards in California By ASCE Los Angeles Section Geotechnical Group Dated June 2002
3. Seismic Hazard Zone Report 58 for the San Jose West 7.5-Minute Quadrangle, Santa Clara County, California, 2002
4. Assessment of the Liquefaction Susceptibility of Fine-Grain Soils By Jonathan D. Bray and Rodolfo B. Sancio, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, September 2006, pp.1165-1177

Dear Mr. Acharya:

In accordance with your authorization, Wayne Ting & Associates, Inc. (WTAI) has completed a geotechnical investigation for the proposed hotel development at the subject site. The purpose of this study was to investigate the site conditions and obtain geotechnical data for use in the design and construction of the proposed development. The scope of this investigation included the following:

a. Review of referenced reports
b. Site and area reconnaissance by the Project Engineer.
c. Two 50-foot deep Cone Penetration Tests were performed by Middle Earth Geo-Testing, Inc. In addition, two additional borings were drilled to a depth of 45 feet to obtain samples for laboratory tests.
d. Laboratory testing of selected soil samples.
e. Analysis of soil samples and information obtained.
f. Preparation and writing of this report which presents our findings, conclusions, and recommendations.

Our findings indicate that the proposed development is feasible from a geotechnical engineering standpoint provided the recommendations in this report are carefully followed.

SITE LOCATION AND DESCRIPTION

The subject lot is relatively flat and located at 1821-1873 Almaden Road, San Jose, California. It is bounded to the east by Almaden Road, north, south, and west by single family structures. An existing Guadalupe River is located approximately 200 to 250 feet southwest and west the subject site. The depth of the Guadalupe River is approximately 22 feet to 23 feet deep. During our site visit, many existing one-story residential structures are presence on the site and will be demolished.

PLANNED DEVELOPMENT

We anticipate that the proposed structure will be 8-story with 1-story underground parking garages. High building loads are typically associated with this type of construction.

FIELD INVESTIGATION

WTAI conducted the field investigation on 17 December and 22 December 2012. The field investigation consisted of a site reconnaissance by the Project Engineer. Two 50-foot deep Cone Penetration Tests were performed. In addition, excavation of two exploratory borings to 45 feet below the existing ground surface. The borings were excavated using a truck mounted drill-rig with an 8.0-inch solid stem auger.

Soils encountered during the excavation operations were continuously logged in the field. Relatively undisturbed samples were obtained by dynamically driving 18 inches using a 3.0-inch outside diameter Modified California Sampler with a 140-pound hammer free falling 30 inches. Blow counts were recorded for every 6-inch penetration interval, and reported corresponding to the last 12 inches of penetration. These samples were then sealed and returned to the laboratory for testing. The classifications, descriptions, natural moisture contents, dry densities and depths from which the samples were obtained, are shown in the Boring Logs, Figures 2 and 3 of Appendix A. The sounding of CPTs are presented graphically on Appendix A.

The locations of the drilled borings and CPT borings are shown on Appendix A, Figure 1, "Site Plan."

LABORATORY TESTING

CLASSIFICATION

The field classifications of the samples were visually verified in the laboratory in accordance with the Unified Soil Classification System. These classifications are presented in the Figures 2 and 3.

MOISTURE-DENSITY

The natural moisture contents and/or dry weights were determined for selected samples obtained during our field investigation. These data are presented in the aforementioned Boring Logs.

UNCONFINED COMPRESSION

Unconfined Compression Tests were performed on three relatively undisturbed samples to evaluate the ultimate compressive strength of the soils. The test results are presented in the Boring Logs.

ATTERBERG LIMITS

The Atterberg Limits Test was determined for the selected soil sample to classify, as well as to obtain an indication of the expansion and shrinkage potential with respect to moisture content variations. The test results are summarized as follows:

Sample	***Depth***	***Classification***	***Liquid Limit***	***Plasticity Index***
B1-1	2.0 feet	Brown silty clay (CL)	39%	18
B2-2	8.0 feet	Grayish brown silty clay	51%	30

The Atterberg Limits tests indicate that a representative sample of the soil is of moderate to high plasticity. The expansion potential for these soils is thus moderate to high.

SIEVE ANALYSIS:

Two samples obtained from Boring 2-5 was obtained for a sieve analysis, the results indicate percent finer than 0.075 mm (sieve no. 200) is approximately 45 percent.

SUBSURFACE SOIL CONDITIONS

The following soil descriptions were derived from our site reconnaissance and the information obtained from our exploratory boring samples. Detailed description of the materials encountered in the exploratory borings and the results of laboratory testing are presented in the Boring Logs. According to Reference 3, the geologic condition at the subject site is located within Qhl. Qhl is the Holocene alluvial levee deposit.

The subsurface soil conditions encountered in our borings at the site were relatively uniform. In boring 1, the site consisted of the site consisted of brown asphaltic concrete and gravel (uncontrolled fills) in the upper 12 inches, followed by medium brown to grayish brown silty clay, firm to very stiff, very moist, to the maximum depth explored of 45.0 feet below the existing ground surface (bgs). A layer of clayey silt and silty sand was encountered between 9.0 and 9.5 feet bgs. In addition, a layer of sandy grave was encountered between 33.0 and 34.0 feet bgs. Ground water was encountered at 32.0 feet bgs.

In boring 2, the site consisted of brown silty clay and gravel (uncontrolled fills) in the upper 2.5 feet, followed by medium brown to grayish brown silty clay, firm to very stiff, very moist, to the maximum depth explored of 49.5 feet below the existing ground surface (bgs). A layer of silt and fine sand mixture was encountered between 23.5 and 24.0 feet bgs. Ground water was encountered at 33.0 feet bgs.

Based on our CPT 1 sounding, interbeded sand, silty sand, and clayey silt was encountered in the upper 3 feet, followed by clayey silt-silty clay and silty clay to 49.5 feet bgs with a few thin layers of sand. Below the clay, very dense sand was encountered to the maximum depth explored of 50.5 feet. It is noted that ground water was encountered at 32.0 feet bgs.

Based on our CPT 2 (Middle Earth show CPT 3) sounding, interbeded sand, silty sand and clayey silt was encountered in the upper 9.5 feet, followed by clayey silt-silty clay and silty clay to the maximum depth explored of 50.5 feet. A layer of silty sand was encountered between 23.5 and 25.5 feet bgs. It is noted that ground water was encountered at 32.0 feet bgs.

In Reference 4, groundwater was encountered at 12.0 feet below the ground surface at the time of the field study. However, according to the ground water data presented on Plate 1.2 of Reference 2, the historic ground water may be approximately 20.0 feet below the ground surface. In addition, according to the CPT test, ground water was at approximately 32 feet below the ground surface. It is noted that fluctuations in the groundwater table are anticipated to vary with respect to seasonal rainfall. Ground water at 20.0 feet will be used for the following liquefaction analysis.

PRELIMINARY LIQUEFACTION EVALUATION

Soil liquefaction is a phenomenon in which saturated (submerged) cohesionless soils can be subjected to a temporary loss of strength due to the buildup pore water pressures, especially as a result of cyclic loadings such as induced by earthquakes. In the process, the soil acquires a mobility sufficient to permit both horizontal and vertical deformations, if not confined. Soils that are most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine sands.

We have conducted a liquefaction for the site based on procedures outline in Special Publication 117A (SP117A), Guideline for Evaluating and mitigating Seismic Hazards in California, Department of Conservation, Division of Mines and Geology. The evaluation procedure is a semi-empirical method for a moment magnitude Mw 7.9 earhtquake, a peak horizontal ground acceleration of 0.51g and groundwater depth of 20.0 feet. In addition, factors of safety 1.3 is used for analysis. Earthquake magnitude and the site acceleration were obtained from Reference 3. We analyze the site liquefaction potential utilizing a computer program call "Liquefypro" by CivilTech; this program is based on the most recent publications of NCEER Workshop and procedure outline in SP117 Implementation (Reference 2).

Based on our analysis using Modified Robertson and Ishihara & Yosemine, it is our opinion that the probability of liquefaction of silt and sand at this site is low. It is noted that the approximately 0.23 inches total settlement and 0.15 inches differential settlements is encountered in CPT test 1. In addition, the approximately 0.55 inches total settlement and 0.36 inches differential settlements is encountered in CPT test 2. The results of the analysis are presented in Appendix B.

A factor of Safety Against Liquefaction (FS) is defined as FS=CRR/CSR

CRR: Cyclic Resistance Ratio represents the liquefaction resistance of the in situ soil.
CSR: A Cyclic Stress Ratio represents the loads induced in the soil by an earthquake.

A factor of safety against liquefaction greater than 1.3 indicates that the level of risk associated with a liquefaction hazard is acceptable.

Lateral Spreading:

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvium material toward an open or free face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soils displace laterally toward the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

It is noted that the Guadalupe River located 200 feet southwest from the subject. Due to potential settlement is small from the liquefaction and sand is encountered in a deep zone below the bottom of the creek, the probability of lateral spreading affecting the site during a seismic event is low. In addition, according to reference 4, the silt and sand mixture encountered in boring 2 between 23.5 and 24.0 feet bgf will not have large deformations as loose clean sand during an earthquake. Therefore, the probability of lateral spreading affecting the site during a seismic event is low too.

CALIFORNIA BUILDING CODE SITE CHARACTERIZATION

The significant earthquakes which occur in the Bay Area are generally associated with crustal movements along well defined active fault zones. According to the published maps by International Conference of

Building Officials (I.C.B.O.), in February 1998, the nearest active fault to the subject site is the San Andreas Fault which is located approximately 17.0 kilometers southwest. Monte Vista - Shannon Fault is located approximately 9.5 kilometers southwest. Hayward Fault is located approximately 17.0 kilometers northeast.. Therefore, the potential for surface fault trace rupture is considered to be negligible. We anticipate that the proposed building will subject to strong ground shaking during the lifetime of the building structure.

The following design values are base on the geologic information, longitude and latitude of the site and the USGS computer program. Furthermore, in according to Chapter 16 of the 2010 California Building Code (CBC), the site seismic design values have been provided as follows:

CBC Category/Coefficient	Design Value
Figure 1613.5.(3), Short-Period MCE at 0.2s, Site Class B, Ss	1.500
Figure 1613.5.(4), 1.0s Period MCE, Site Class B, S1	0.600
Table 1613.5.2, Soil Profile Type, Site Class	D
Table 1613.5.3(1), Site Coefficient, Fa	1.0
Table 1613.5.3(2), Site Coefficient, Fv	1.5
S_{MS} = Fa x S_s Spectral Response Accelerations	1.500
S_{M1} = Fv x S_1 Spectral Response Accelerations	0.900
S_{DS} = 2/3 x S_{MS} Design Spectral Response Accelerations	1.000
S_{D1} = 2/3 x S_{M1} Design Spectral Response Accelerations	0.600
** Latitude 37.301868, Longitude: -121.880115	

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

1. Based on the results of our investigation, WTAI concludes that the subject site is geotechnically suitable for the proposed development. The proposed building can be constructed provided the recommendations presented in a final report are incorporated into the project plans and specifications. The final report may include recommendations for excavation of basement, site grading, foundation, basement wall, concrete slabs-on-grade, pavement design, utility trench backfill, and surface drainage.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

2. Our client should recognize that the conclusions and recommendations reached in this report are based on soil samples recovered from the test boring locations and results of laboratory tests on selected soil samples. These soil borings are not necessarily representing conditions at other locations. Our geotechnical study has been conducted in accordance with current professional engineering principles and practices. No other warranty is expressed or implied.

3. The conclusions and recommendations contained in this report will not be considered valid after a period of two years unless the changes are reviewed, and the conclusions of this report are modified or verified in writing.

Should you have any questions relating to the contents of this report, please contact our office at your convenience.

Very truly yours,

WAYNE TING & ASSOCIATES, INC.

Wayne L. Ting, C.E.
Principal Engineer

Copies: 3 to Mr. Acharya

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APPENDIX A

Site Plan, Figure 1

Boring Logs, Figures 2 and 3

CPT Logs